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February 9, 2000

FEB 28 2000

Ms. Wendy R. Dixon
EIS Project Manager M/S010
US Department of Energy
Office of Civilian Radioactive Wastes Management
Yucca Mountain Site Characterization Office
PO Box 30307
North Las Vegas, NV 89036-0307

Dear Ms. Dixon:

1 I spent four hours at the hearings held on February 1 at the Hotel Intercontinental in Chicago. The whole process was beautifully handled by your staff and consultants.

2 I am attaching some information that I have prepared previously, which is not in professional form, but certainly expresses the ideas that I feel relevant.

It seems to me that each of the 77 nuclear power plant sites, 72 private and 5 government, should have on their own properties, temporary storage for 100 years, termed a "spent rod safe." It's characterized on the drawings and discussed herein. In my opinion, the idea will not work if you rely on traditional system of getting competitive bids to have the work performed.

The parties to the project, engineering, construction, including the subdivisions of construction, should be picked based on their competence and experience, and integrity. There should be no problem in negotiating a fair price. If some of the firms mentioned in my write-up are contacted. I have already talked to them.

The system would take care of the immediate needs of each of these power plants, and a safe environment protected from terrorism, permitting reclaiming the capsules at will, triple protecting the ground water and ground with triple casing, and with monitoring systems that will use the latest Motorola systems for monitoring conditions inside the Spent Rod Safe, as well as in the soil surrounding the "Safe."

If you do this expeditiously, then the pressure on the Yucca Mountain site would be gone.

3 Personally, it is my opinion that it's a bad idea to build the Yucca Mountain site. Why we should accumulate all the nuclear waste at the same site is beyond my engineering common sense in the first place. Somebody at the hearings today, February 1, discussed an effort in Siberia some years ago when Russa put all their wastes into some trenches. They reacted with each other and exploded, killing 1,000 people.

I am chairman of ASCE standard committee on design and analysis of nuclear safety related earth structures, a copy of which standard is attached. This is the first ASCE standard ever produced.

I was also vice chairman and acting leader of a group under the auspices of the National Academy of Sciences, who investigated the radioactive waste disposal on the Enewekok Atoll, 1980-82.

I was also a consultant to Holmes and Narver when they were first planning to isolate underground tunnels from vibrations due to test blasts near the Nevada test site tunnels.

Also, I was the first one to use nuclear soil moisture and density meters in construction, a process that was developed at Cornell, where I graduated in 1946 in structural engineering.

4 If I were to summarize the discussions on February 1, I would have to say that almost all of the preoccupation of those who objected to the whole system were unhappy about the transportation of the spent rods through the public highway system. To dispose of the spent rods temporarily for 100 hundred~~s~~, or as one wishes to look at it, it would eliminate the concern of most citizens.
years

The biggest thing, however, is the cost savings as well as the lack of inconvenience to the public who object to the spent rods on the highways.

Even aside from the protection of the environment, nuclear power also is a continuing supply, compared to coal, oil, and gas which have finite limits, aside from the damage, particularly coal, to the environment.

I hope these comments are useful and would be most pleased to meet with you or somebody you designate to talk about these ideas on a moment's notice.

will send

I would be quite pleased if the ideas were accepted as being public property, though I would appreciate being considered as a consultant in planning actual applications to different nuclear power plants. There are altogether too many people in the foundation consulting business and foundation construction business who need independent project peer review. You might be interested in the attached standard on independent project peer review, prepared by a committee of which I am chairman, which should be a requirement for any projects that are done by DOE, particularly the spent rod storage silos.

I hope these ideas are useful.

Sincerely,

John P. Gnaedinger
Registered Structural Engineer in Illinois

Spent Rod Safe™
by John P. Gnaedinger

There continues to be a national debate, and local reluctance to place spent rods, in the Yucca Mountain cavern design system. While there appear to be some very strong political pressures, not logic, as to the selection of this site as a national repository for spent rods, the local objections are even stronger.

5 The problems of putting all the spent rods in one site seems in itself to be counterproductive. After all, nuclear energy was developed in the first place by bringing the nuclear rods closer and closer together, under the Stagg Field bleachers at the University of Chicago.

The transportation of spent rods from all of the nuclear power plants in the country, some 60 total perhaps, involves substantial risk and cost.

There have been several shipments that overturned when their semi's were in accidents or otherwise had problems, exposing local people to some risk. The cost of the capsule is, to my knowledge, of the order of $1 million, involving the same 9" steel protection that is used for containment vessels at nuclear power plants themselves. It is also true that there are many alternatives being used for the design at these transportable capsules by different producers.

The Yucca Mountain site has two other issues involved with it. The first is that the Supreme Court has decided that the US Government must find a disposal site for spent rods, in terms of the 1982 legislation regarding nuclear power. The Federal Government has been frustrated in trying to find somebody willing to take the spent rods into their state, much less their community. They couldn't even reach agreement on a site to locate low level nuclear waste.

Thus, the Federal Government at the moment is faced with a decision as to what to do about their dilemma. The ideas proposed herein could represent a solution to that dilemma. It would cost the Federal Government perhaps of the order of 1% as much to pay for the cost for locating these spent rods in temporary storage at the site of the nuclear power plants themselves, rather than hauling them anywhere.

6 Another factor involving Yucca Mountain is that power companies have been paying one mil per kilowatt hour into a fund to complete the Yucca Mountain project. While it appears that these funds are perhaps being wasted, demonstrating the solution to the problem long before the site has been approved, which seems imprudent, there has also been a suggestion that actually the sums paid into the fund are of the order of $50 billion, and most of these funds might have been used for other government expenditures unrelated to the purpose of the funds, namely to handle spent rods.

7 There is another issue, perhaps more entrepreneurial than in political, that the power companies themselves should be the ones to accept responsibility for the disposal of the rods, particularly since they at least can dispose of them on their own property, as they are presently doing as a matter of necessity.

It seems to me that the Federal Government not only should support these techniques, but perhaps under the commitment of the 1982 Nuclear Act, should pay the cost. They certainly have enough money already on deposit in their Yucca Mountain fund to pay all these costs.

The ideas presently being used includes actually keeping the spent rods in above grade swimming pools at most of the nuclear power plants in the country. This technique does expose the spent rods and their storage facilities to a risk of Oklahoma City or World Trade Center terrorist practices and actions. It would therefore be much safer to have the spent rods buried, as is the proposal of the Spent Rod Safe™ concept.

A patent was obtained five years ago for storing the spent rods in ammunition bunker type structures, presumably at or in the vicinity of the nuclear power plant.

8 Outside storage on compacted granular pads, using stainless steel lined capsules, is being implemented by some firms, including Northern States Power.

Again, there is some exposure to vandalism or worse in terms of this outside storage.

Spent Rod Safe™ Concept

9... The inventor of this concept has been involved in the installation of the first deep caissons supporting major structures in this country. The installation of these caissons originally utilized equipment manufactured by Casey & Case in Los Angeles. Through the years, machine-drilled caissons supporting major structures in Chicago have replaced the traditional hand drug caissons, which have been used in Chicago for almost 100 years. The machine dug caissons have a great advantage cost-wise, and also safety-wise, both with regard to workers on the project, and with regard to the adjoining structures, providing proper precautions and steps are taken during installation of the caissons, of course.

The equipment is currently capable of drilling through soil or rock with diameters up to 11', and with depths up to 500'. The equipment for the rock drilling is similar to equipment used for tunneling, but operated vertically, in a mudded drill hole.

9 cont. The concept herein is that the rods be stored in capsules that are placed in 100 year temporary storage on the inside of perhaps a 7' diameter stainless steel casing at the center of the installation. The 7' steel casing would be surrounded with an 8' steel casing, with the annulus between the two filled with boron frit, recognized for its ability to absorb certain radiation.

The outside casing would be presumably 9' in diameter, regular steel, which is more resistant to deterioration due to soil chemicals than is stainless steel. The casings could further be protected by polyurea coatings, inside and out, of the order of 125 mil thickness.

In constructing the installation, there would of course be a drilling operation using mud, to drill to the desired depth of perhaps 500, or even 1000'.

Then the external casing would be progressively lowered in the mudded shaft, with a special rigging at the surface to prevent it from dropping, and with proper welding at the top of each casing when the next section is added.

Perhaps this could be accomplished more safely by having the casing welded together to perhaps 100' total length, before attaching to the casing already in the hole, to minimize the number of welds that would be made with a hanging arrangement for the casing being assembled.

It is proposed that in view of their top expertise in welding, such as with liquified natural gas tanks, that Chicago Bridge and Iron would the firm who would be involved in doing this welding.

After the welded assembly 500' long reaches the bottom of the shaft, then the bottom would be cleaned off with reverse rotary. An "airlift" system would work beautifully. Concrete would be tremie placed to perhaps a 50' depth at the bottom, as a plug in the bottom of the shaft. Perhaps a lesser depth would suffice.

Flotation would, of course, be a condition, to be avoided when the shaft is dewatered.

For many reasons, it might be preferable to construct a crane structure around the areas of proposed storage, <u>before</u> the shaft is drilled, with perhaps an arrangement for doing the drilling, installing the casings, and doing other work inside, to expedite the project and also to prevent vandalism for later purposes.

The slab of the proposed structure might be constructed of a 4-'6" thick concrete slab, with provision to anchor the external casing into this very heavy slab, to prevent flotation when the shaft is dewatered.

9 cont. Certainly different contractors would have different ways of handling the shaft drilling. It is proposed that Case Foundation Company, who has done much of the major caisson construction in the midwest, and elsewhere, in many cases, would be responsible for installing the shaft. They have given an estimate of $2.6 million as of September 1998, for installing two 500' deep shafts at a site, not counting steel costs or the installation of the casing.

It is expected that since the spent rods are changed every 18 months, and one capsule, perhaps 15' long would be required, at each change, that 100 years would require 1,000' of hole. Thus, the two 500' holes would suffice for each power plant, though the actual number, diameter and depth would depend on the number of units at a power plant and the other factors that would vary from site to site.

It is anticipated that Automated Engineering Company, Dr. Achyut Setlur, would be responsible for the design of the installation.

It is also assumed that STS Consultants, Ltd. would be responsible for the coordination with the contractor on the installation, and for inspection and instrumentation related to the installation.

It is, of course, expected that there would be substantial nuclear radiation instrumentation in the outer annulus, as well as ground water monitoring installation that would be read continuously, with equipment perhaps provided by Motorola, Inc.

Even if the building discussed is installed after completion of the shaft, to facilitate shaft performance, the building would still be beneficial after completion as a security measure, to essentially eliminate any access to the site or any opportunity for vandalism or other damage because of a security system and a surface plug at the top of the shaft that would not be easily removed.

It would also be earthquake proof, providing of course, it is not installed in an actual fault. Even in an area that might have earthquakes, the shaft would move with any earthquake movement without damage.

The capsule would be retrievable, either by a one by one retrieval system, or perhaps by having stainless steel cables so installed that each unit could be lifted as required, for recovery of the spent rods materials to later facilitate production of nuclear power, or for other reuse.

9 cont. It is further proposed that all contracts on the project would include provision for Independent Project Peer Review, in accordance with ASCE Standards 22-97. Further, any disputes involved with construction would be resolved by mediation/arbitration, with a five person mediation arbitration board preselected by all parties in advance of construction. This would provide final and binding resolution of any disputes that are not resolved by negotiations and discussions, facilitated with good communication on the project.

Since the entire operation would take place at the nuclear power plant site itself, it would substantially reduce any risk to the neighborhoods, of nuclear problems from the spent rods. Because present storage is in swimming pools, it may not be necessary to get additional approval on installation. But it should be an easy matter to convince those to whom the approval would be presented, that this would represent a substantial increase in safety to the power plant, to the community, and to the country by eliminating the Yucca Mountain $250 Billion cost and related risks.

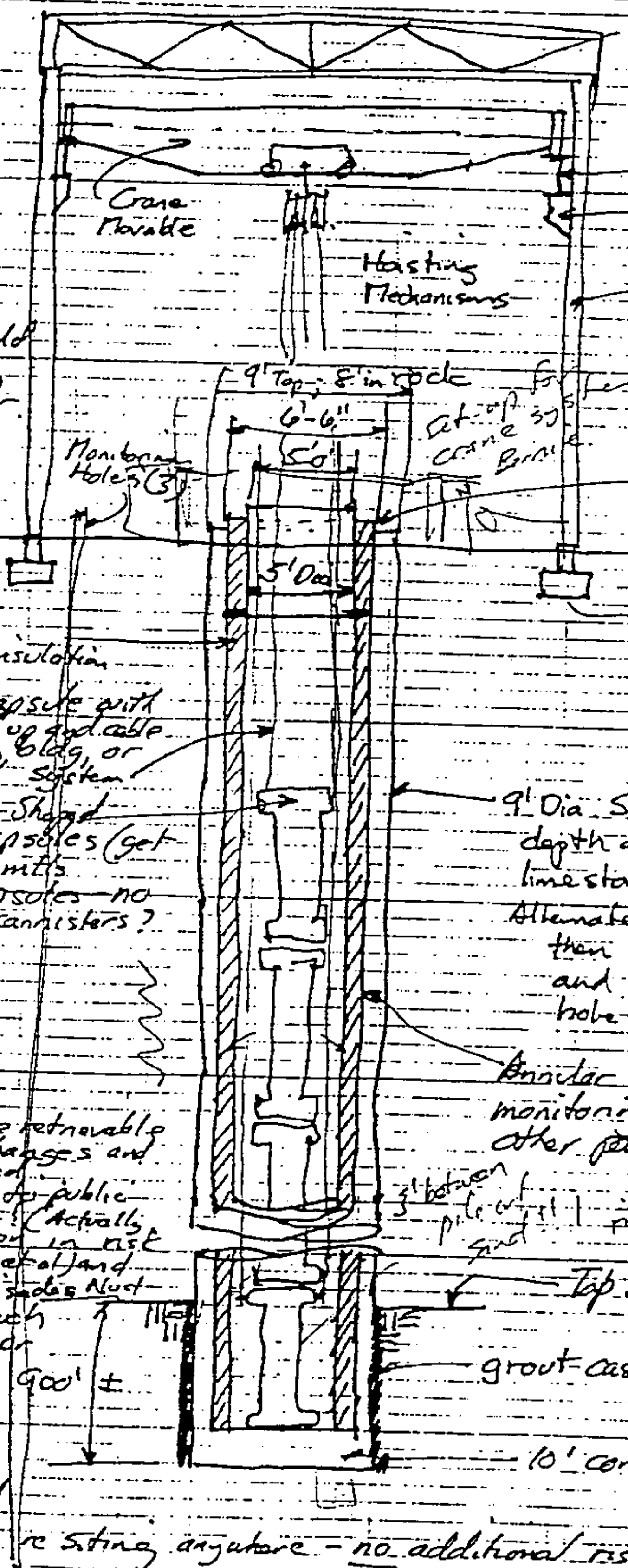
Respectfully submitted,

John P. Gnaedinger
President, John P. Gnaedinger Research Corporation
December 11, 1998

STS Consultants Ltd.
CALCULATION SHEET

PROJECT	On-Site Burial of Nuclear Wastes
STS JOB NO.	JPG
SUBJECT	Sketch of Concept e.g. Zion N.P.P.
CALC. NO.	
DATE	5/16/94
BY	JPG
CHECKED BY	
REV. NO.	

JPG Note:
Many improvements possible.
Only experienced contractors should even be called. Could be mine shaft contr, drilled caisson contr.



Notes –
1. Capsules would be retrievable if technology changes and wastes can be reused.
2. No increase in risk to public from present system! Actually substantial reduction in risk from pools (Dresden et al) and Containers at Palisades Nucl.
3. No public outcry such as at Yucca Flats or even Martinsville IL
4. Constant Monitoring easy – and action if required
5. No public risk – no transporting req'd
6. No public outcry re siting anywhere – no additional risk over NucPwrPl itself

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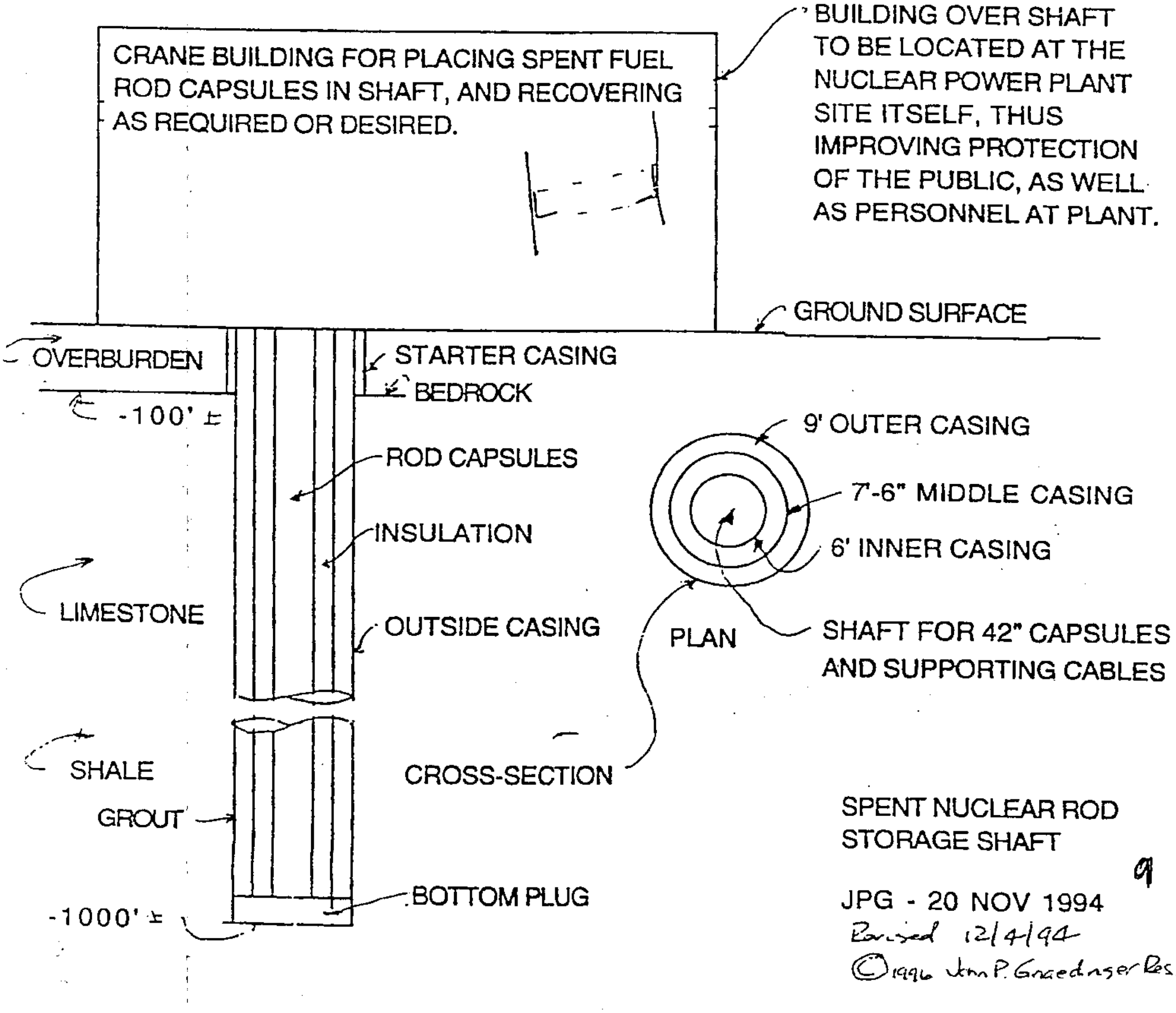
SPENT NUCLEAR FUEL ROD STORAGE FACILITY

Risks to the public involved with transportation of spent rods to Yucca Flats can be eliminated by deep storage in triple cased shafts at the nuclear power plant site itself. The concentration of such radioactive wastes at one location, Yucca Flats, seems to be, in itself, a magnification of risks to the public. On the other hand, to take such spent rods, which are now stored in temporary pools of water at each site, and lower them into deep shafts, would increase public safety thereby, at a cost many orders of magnitude less than the Yucca Flats burial cavern concept.



SPENT NUCLEAR ROD STORAGE SHAFT

JPG - 20 NOV 1994

Revised 12/4/94

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November 5, 1986

ASCE STANDARD

N-725 Guideline for Design and Analysis of Nuclear Safety Related Earth Structures

OCTOBER, 1988

AMERICAN SOCIETY OF CIVIL ENGINEERS

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N-725 Guideline for Design and Analysis of Nuclear Safety Related Earth Structures

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FOREWORD

For earth structures, it is recognized that the use of nominal material parameters and design assumption alone is often inadequate. Coupled with these should be site verification of design assumptions and materials parameters, evaluation of the interrelationship between construction methods and analytical treatment, and verification testing (to assure compliance with project specifications based on the design and monitoring of the earth structures—both during and subsequent to construction). Because of the need for continuity from design and analysis through actual construction, Section 7.0 (which deals with inspection, instrumentation and monitoring) is included in this standard, although the standard itself is primarily concerned with design and analysis.

Sampling, testing and interpretation of data, appropriate recognition of results and their limitations, and recognition of the interrelationship of the many sets of data; characteristics and other parameters are crucial to the proper performance of the design and construction discussed in this standard.

It is for these reasons that responsibility for the design and construction of safety-related earth structures rest only with engineers who are experienced in one or more of the various phases of earth structures design and construction (including soil and rock mechanics, geology, field sampling, laboratory testing, analytical methods, specifications, construction control and instrumentation). Such engineers shall also have adequate understanding of the hydrological aspects of impoundments, fluid motion effects and of seismological inputs to the site.

This standard includes administrative requirements dealing with responsibilities and peer review, as well as scope, in Section 1.0. Section 2.0 includes a reasonable breadth of definitions to make the standard usable for individuals knowledgeable in the geotechnical and foundation design field.

Site investigations cannot in themselves be standardized with regard to application to a particular project because of the wide range in soil and rock conditions that exist in nature, and the innumerable design alternatives that are typically present for the design of earth structures. Therefore, informative references have been included in Section 3.0 on Site Investigation, with additional pertinent references in specific sections of the standard.

The essential elements of the standard are:

(1) Section 4.0 covers earth structures used to form the ultimate heat sink (reservoirs) including dams, dikes, and baffles;

(2) Section 5.0 covers earth structures normally used to protect the nuclear plant site from extreme hydrodynamic loads including dams, dikes, breakwaters, seawalls, and revetments;

(3) Section 6.0 covers earth structures used to maintain site contours, the stability of natural and cut slopes, fills and retaining walls.

Section 7.0, Inspection and Monitoring, relates to unique problems associated with geotechnical site conditions and construction necessitating a close relationship between geotechnical studies and design, construction aspects and related monitoring, and onsite verification of actual conditions encountered with regard to original studies and design assumption. Because variability of geotechnical conditions from point to point on a site is expected, the geotechnical study cannot be completed until the project is completed, with monitoring providing additional design verification.

Earth structures involve unique construction processes for each project. And soil conditions present a contrast to many elements of nuclear power plants that are manufactured (and can be held to narrow limits of variability and tested for approval within those limits). This prevents, in this standard, use of restrictive or definitive requirements that might be entirely feasible for other aspects of nuclear power plant construction.

Natural soil and rock conditions are subject to considerable variability. This element is not typical of the kind of structural problems inherent to nuclear power plants. Such natural variability must be recognized. There are also numerous analytical procedures available, some of which may be particularly applicable to particular soil or foundation conditions while not applicable to others. For these reasons, Section 1.2.3 on Peer Review is incorporated with this standard. It is recommended as the process most likely to minimize major problems and at the same time minimize overdesign.

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N-725 GUIDELINE FOR DESIGN AND ANALYSIS OF NUCLEAR SAFETY RELATED EARTH STRUCTURES

1.0 Purpose, Scope and Administrative Requirements

1.1 Purpose and Scope. The purpose of this standard is to describe parameters and present guidelines and criteria to be used in construction of those earth structures forming part of the ultimate heat sink or act to protect nuclear power plant sites from flood, storm surge or other types of natural or manmade extreme load phenomena. Such structures are identified as safety related (Seismic Category I[1]) structures. Also included are earth structures required to maintain finished grade and ground contours at nuclear power plant sites.

Construction includes all administrative, quality assurance and regulatory requirements, material selection, design, installation, examination and monitoring associated with the safety related earth structures. Included in this standard are requirements for earth dams, dikes, baffles, breakwaters, seawalls, revetments, cut and natural slopes, retaining walls, and compacted fills, whether of natural or cement stabilized materials. It includes identification of design margins of safety.

1.2 Administrative Requirements

1.2.1 Responsibilities. The Owner shall prepare a Design Specification for earth structures which shall define the basis of design and other applicable criteria. The Design Specification shall include the following:

(1) the function and boundaries of the earth structure
(2) the specific load requirements (including load combination) to be considered in design of the earth structure
(3) behavior and operational requirements for the earth structure
(4) the design life
(5) acceptance testing requirements (if any)
(6) monitoring and construction process review required of the Geotechnical Engineer.

The Geotechnical Engineer shall have the responsibility of preparing the Construction Specification as described in Section 7.3. In accordance with the scope of this standard, several of the above activities are discussed herein as they pertain to the construction of nuclear safety-related earth structures.

1.2.2 Quality Assurance. Quality Assurance programs that comply with the Code of Federal Regulations 10 CFR 50, Appendix B,[1] shall be established with regard to all elements in the design and construction of safety related earth structures of nuclear power plants. The design, inspection, and review services performed by the owner or his agent do not relieve a contractor of the responsibility for performing the work in accordance with the criteria, plans and specifications and applicable regulatory requirements. Each contractor shall retain responsibility for the quality control of his own services and workmanship.

1.2.3 Peer Review

1.2.3.1 Initiation of Review. Independent reviews shall be conducted by the Owner to provide assurance that the quality of safety related earth structures is in accordance with the standards of the profession, that objectives of the work are met and the safety of the public is provided for. The owner should select one or more peer reviewers and define the depth of the review.

1.2.3.2 Qualification of Reviewer. Peer reviewers shall possess the technical qualifications, practical experience, and professional judgment required of the work to be reviewed.

1.2.3.3 Review Requirements. Peer review shall include a critical independent assessment of the validity of the following items as used in the development of the design:

(1) design basis loads
(2) material properties
(3) design concepts
(4) methods of analysis
(5) safety factors.

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It does not necessarily include a complete check of detailed calculations. Upon completion of the review, peer reviewers shall present their findings to the originator of the work and the Owner.

1.2.3.4 Documentation. The results of the peer review are documented by the reviewer. The record of the peer review identifies the reviewer, the scope of the review, and all pertinent findings concerning the project.

2.0 Definitions

The following definitions are provided to assure a uniform understanding of some selected terms as they are used in this standard.

Baffle—Any earth, rock or earth/rock constructed barrier used to control flow or direction of water (such as used in cooling reservoirs). A baffle may be partially or totally submerged.

Breakwater—A massive offshore structure constructed of rubble, rock, and/or performed armor units designed to dissipate wave energy before a wave reaches shore or to control intake or discharge water flow paths to prevent recirculation.

Bulkhead—Any permanent continuous wall containing wood, masonry, steel or concrete members driven or installed for the purpose of retaining an earthbank, water or other materials.

Category 1 Structure—(See Safety Related Earth Structure).

Construction Process Review—A review to verify that the necessary assumptions made in the design and analysis of the earthwork are compatible with the actual construction method and sequence.

Construction Process—All those activities (including conformance with applicable regulatory or administrative requirements, design, material selections, fabrication, erection and examination) necessary to construct the earth structures of a nuclear power plant.

Construction Specification—A document prepared by the Geotechnical Engineer, which contains sufficient detail to provide a complete basis, including inspection, test or monitoring requirements for construction of earth structures in accordance with the applicable Design Specification.

Contractor—An organization under contract to furnish items or services related to the construction of earth structures.

Criteria—Parameters which must be complied with by a design, analysis, or by construction procedures or results.

Dam—Any earth, rock or earth-rock constructed barrier which, together with appurtenant works, impounds water.

Dike—Any earth, rock or earth-rock constructed barrier which contains a canal, stream or other water carrying channel or which restrains the flow of water or other liquid after a failure of another impounding barrier (such as a dam or tank). Also, massive placement of soil, stone or rock fill designed to protect nuclear plant sites from flooding or to otherwise channel or divert water away from a site.

Engineer—A person or organization having design and analysis responsibility for construction of a nuclear power plant.

Examination—A phase of quality control which, by means of observation of measurement, determines conformance of structures to predetermined quality standards.

Geotechnical Engineer—A person or organization competent and recognized as knowledgeable to provide services dealing with soil and rock mechanics and foundation engineering.

Geotechnical Services—Services performed by a geotechnical engineer.

Monitoring—(see Verification).

Operating Basis Earthquake—(OBE)—The earthquake whch, considering regional and local geology, seismology and specific characteristics of local subsurface material, could reasonably be expected to affect the plant site during its operating life. It is that earthquake that produces the vibratory ground motion for which those features of a nuclear power plant necessary for continued operation (without undue risk

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to the health and safety of the public) are designed to remain functional.

Owner—The company or corporation who has, or will have, title to the facility of installation under construction.

Peer Review—A review by a peer(s) of the originator of the work to assure that the quality of the geotechnical work is in accordance with the standards of the profession and that objectives of the work are met (see Section 1.2.3).

Performance Monitoring—Activities conducted to define the state of the completed work relative to design assumptions. Monitoring may be prior to, during, or subsequent to construction.

Quality Assurance—The planned or systematic actions necessary to provide a means to control and measure the characteristics of an item, process or facility to established requirements.

Retaining Wall—Any permanent structural element built to support a vertical, or near vertical, earth bank to retain water or other materials.

Revetment—Facings of stone, precast armor units etc., built along the waterfront to protect a scarp, embankment, or short structure against erosion by wave action or currents. They are built to protect against direct wave attack by absorbing wave energy in their interstices and on their surface.

Safe Shutdown Earthquake (SSE)—That earthquake based upon evaluation of the maximum earthquake potential, considering regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which those structures, systems and components important to safety are designed to remain functional.

Safety Related Earth Structure—Earth structures necessary for required safe operation of systems necessary to the safe shutdown of the nuclear plant, engineered safeguards or to control release of radioactivity.

Seawalls—Massive structures along the waterfront of rubble, rock, stone, concrete and/or preformed armor units designed to prevent waterfront erosion or damage due to wave action, and designed to take full impact of the design wave.

Slope, Natural—An inclined soil or rock surface resulting from natural geologic processes.

Slope, Cut—An inclined soil or rock surface resulting from excavation of soil and/or rock.

Specification—A concise statement of the requirements or criteria to be satisfied by construction indicating the procedure by which it may be determined whether given requirements are satisfied.

Standard—Result of a particular standardization effort approved by a recognized authority. Standards include Codes and Guidelines.

Testing—Determination or verification of the capability of an item to meet specified requirements by subjecting it to a set of physical, environmental or operating conditions.

Ultimate Heat Sink—That body of matter, usually liquid, which absorbs heat initially generated in a nuclear reactor core and which is necessary to keep the temperature of the reactor core within specified limits.

Verification—An act of confirming or substantiating that an activity or condition has been implemented in accordance with specified requirements.

Verification, Design—Verification of assumptions, analytical methods, and design concepts used in the design analysis.

Verification, Excavation—Verification that soil and rock conditions in excavated areas are consistent with conditions assumed when preparing earth structure designs.

Verification, Geologic—Verification that exposed soil and rock conditions are consistent with conditions shown on boring logs and profiles.

3.0 Site Investigation

Site investigation shall consider geology, topography, seismology, materials availability, and design function. The seismological and geological investigations for these structures shall be consistent with other safety related structures at the site.

Each section of this standard discusses site investigations to identify special considerations in performing such work. However, at the end of this Section 3.0 are identified reference materials on site investigations, including laboratory testing, that are generally applicable.

Geophysical exploration methods such as seismic refraction, reflection, and electrical resistivity should be used to locate ground water table, faulting, and determine depth to bedrock (if applicable). The subsurface exploration program should consist of borings, test pits, trenches or inspection shafts to reveal critical stratification, ground water table and obtain representative and undisturbed test samples.

Laboratory testing to determine soil parameters should include standard classification tests, strength tests on undisturbed samples and consolidation testing (if appropriate). In situ strength tests to determine strength parameters are also recommended. Static or dynamic Dutch cone penetration test (CPT) and standard penetration tests (SPT) should be considered to qualitatively evaluate in situ densities of cohesionless soils for correlation with static and dynamic parameters. A qualitative measure must employ a site determined correlation. The ground water table level shall be recorded in selected boreholes, with sufficient time allowed for stabilization of the water level. Any data relevant to the variability of the ground water table and the source of variation should be investigated.

Of particular importance are:

ANSI N 174 "Guidelines for Evaluating Site-Related Geotechnical Parameters for Nuclear Power Sites," Prepared by ANS Committee 2.11, ANSI, 1978

ASCE "Subsurface Investigation for Design and Construction of Foundations of Buildings" Manual No. 56, 1976

ASTM Book of Standards, Part 19, "Natural Building Stones; Soil and Rock; Peats, Mosses, and Humus"

ASTM "Special Procedures for Testing Soil and Rock for Engineering Purposes," STP 479

NRC Regulatory Guide 1.132 "Site Investigations for Foundations of Nuclear Power Plants," U.S. Nuclear Regulatory Commission Office of Standards, Sept. 1977

NRC Regulatory Guide 1.135 "Normal Water Level and Discharge at Nuclear Power Plants," U.S. Nuclear Regulatory Commission Office of Standards, Sept. 1977

ANSI N 45.2.20 "Supplementary Quality Assurance Requirements for Subsurface Investigations Prior to Construction Phase of Nuclear Power Plants," American National Standards Institute, 1979

ANSI N 45.2.5 "Supplementary Quality Assurance Requirements for Installation, Inspection and Testing of Structural Concrete, and Structural Steel, Soils and Foundations During the Construction Phase of Nuclear Power Plants QA-76-5" 1978

Code of Federal Regulations 10 CFR 100 Appendix A "Seismic and Geological Siting Criteria for Nuclear Power Plants," U.S. Atomic Energy Commission, November 1973.

4.0 Ultimate Heat Sink Earth Structure—Dams, Dikes, and Embankments

4.1 Scope

4.1.1 Purpose. The purpose of this section is to describe parameters and to present guidelines and criteria to be used in construction of ultimate heat sink structures, and to identify factors which should be considered throughout their conception, siting, design, and operation.

4.1.2 Use and Type of Structures. This section includes earth structures, which are a means of water conveyance, impoundment, diversion or control. These include but are not limited to the following:

(a) cooling water supply reservoirs
(b) essential cooling ponds
(c) essential heat sinks
(d) waste-water retention structures
(e) flood-protection dikes and levees

The maintenance of water retaining function is the prime consideration in the application of these structures.

4.2 *Site Investigation*. A general discussion of site investigation applicable to all earth structures is presented in Section 3.

4.2.1 *Seismology and Geology*. General seismic siting criteria are given in 10 CFR 100, Appendix A.[1]

Various other references provide useful information on the requirements, which must be satisfied by a thorough seismologic and geologic investigation.[9, 10]

4.2.2 *Hydrology*. Structures in combination with their appurtenant works (spillways, overflow sections, etc.) shall be designed to withstand historical and design basis floods as determined in accordance with ANSI N 170.[11]

4.2.3 *Geotechnical*. In the construction of earth structures, the structure cross section, materials of construction and their graduation, zoning and placement shall be consistent with site geology and foundation conditions. Investigations shall be undertaken and sufficient information obtained so that the engineer can design a structure which meets those requirements. References that discuss required geotechnical investigations in considerable detail should be consulted.[11, 12, 13, 14, 15, 16, 17]

4.3 *Materials*. The Geotechnical Engineer shall verify that materials used, and the specified manner in which they are used and placed, are compatible with the design. References that discuss selection of materials and appropriate cross sections and zoning include references 11 and 12 through 19.

Locally available materials may be used if they are appropriate. The embankment should be properly zoned to provide the following:

(a) an impervious zone
(b) transition zones between core and shells
(c) seepage control
(d) static and seismic stability
(e) wave protection.

Laboratory tests shall be conducted to evaluate required characteristics of various materials to be used in construction of embankments; these include classification tests and tests to evaluate gradation, compaction, strength and compression characteristics of the various types of materials.[12, 15, 20, 21, 24]

4.4 *Design*

4.4.1 *Design Parameters.* Parameters to be established for the design and safety evaluation of dams, dikes and baffles shall include the following:

(a) a geotechnical profile along the entire length of the structure foundation and across the structure foundation at ¼ the width in equal intervals, or more, in order to provide a basis for design
(b) soil properties sampled and tested under anticipated environmental and loading conditions including strength, compressibility, permeability and durability
(c) the potential for ground surface rupture or displacement due to geologic factors
(d) ground surface vertical and horizontal acceleration and damping coefficients for the SSE
(e) the design depth of water for the structure
(f) the height, length and period for the design wind = generated wave
(g) the characteristics of the maximum probable wave which could impinge upon the structures (i.e. average of highest one percent of all waves, H_1, or tsunami, or dam = break wave[11])
(h) properties and qualities of available cast shapes, rubble, stone, rock and filter materials used for construction of the structure
(i) cross sections showing structure geometry and composition of materials
(j) liquefaction potential of structure/soil foundations under (a) the SSE and (b) hydrodynamic changes in effective stress
(k) stability of the structure and its foundation under all design loading conditions (including hydrodynamic force systems associated with the SSE)

(l) ability of the structure to withstand continual hydrodynamic forces without relative movement of its internal components, which are sufficient to cause structural failure.

4.4.2 *Operating Conditions.* Operating conditions for impoundments will vary according to purpose, location (on-stream or off-stream) and other conditions unique to the plant being considered. These conditions may influence design of the structure as well as loading conditions, factors of safety-slope protection, materials of construction, zoning, seepage analyses, and other parameters. They may influence the design of ancillary facilities. The Geotechnical Engineer shall consider all normal operating conditions in design of the structure, as well as anticipated transients, abnormal and extreme environmental conditions, which are considered as design basis during the life of the structure (as defined by the Owner in the design specifications).

4.4.3 *Static Loading Conditions.* The following conditions shall be considered for dams and dikes:

(1) During construction
(2) End of construction
(3) Sudden drawdown from spillway crest to minimum pool evaluation: This may not be necessary if size of outlet or other passive means does not permit sudden drawdown. The relative permeability of the dam's upstream material and the potential rate of the maximum drawdown should be considered.
(4) Sudden drawdown from top of spillway gates to crest of spillway (if any), if such a condition could occur.
(5) Full reservoir or partial pool, downstream slope, steady seepage: The critical case should be determined through a parametric study of the factors influencing the selection of condition. Generally, the full reservoir case will govern unless it is an assured temporary condition. Steady seepage with a reservoir surcharge may fall into this category.
(6) Sudden drawdown on downstream slope: This case may occur where the downstream toe is subject to prolonged flooding and then rapid reduction of the toe water level. This case will not normally be critical where the downstream toe is relatively porous.

4.4.4 *Static Stability and Performance*

4.4.4.1 *Dams and Dikes.* Factors of safety for embankment stability studies should be based upon the ratio of available strength to applied stress or other load effects. The minimum factors of safety for the static loading conditions listed in Paragraph 4.4.3 shall be as follows:

Condition	Minimum Factor of Safety
1	1.1
2	1.3
3	1.0
4	1.2
5	1.5
6	1.2

In using these minimum recommended safety margins the Geotechnical Engineer should have a high degree of confidence in the reliability of the values used for the following parameters:

(a) type and gradation of material (identification)
(b) thoroughness and completeness of field exploration and laboratory testing (performance of materials)
(c) loading conditions
(d) degree of control and workmanship expected.

4.4.4.2 *Baffles.* For baffles (or dams which may be submerged), the fully submerged and drawdown conditions shall be considered. The effects of the failure of an earth structure upon the containing dike shall also be considered. Consideration shall be given to the flow of water through and over the earth structure. The minimum factor of safety of the baffle and its containing dike (or dam) shall be the same, or greater, as for the dike (or dam) itself.

4.4.5 *Dynamic Loading Conditions.* The effects of earthquake-induced forces, currents, floating debris, and wave action on behavior and peformance of safety class earth dams, dikes and baffles must be considered. The postulated failure conditions due to a dynamic load to be evaluated are as follows:

(1) Failure due to disruption of the structure by major differential fault movement in the dam foundation.
(2) Slope failure induced by SSE vibratory ground motions.
(3) Sliding of structures on weak foundation materials or materials whose strength may be reduced by liquefaction.
(4) Piping failure or seepage through cracks induced by ground motions.
(5) Overtopping of the structure due to seiches in the reservoir, slides or rock-falls into the reservoir or failure of the spillway or outlet works.

Other dynamic-induced forces to be considered in design are:

(a) transfer of momentum effects from moving currents at design maximum flood condition
(b) impact of any postulated floating missiles at design maximum flood condition
(c) design wave load effect (including the effect of wave frequency and momentum).

In general, failure mode (1) is precluded by siting restriction. While earth structures tend to be able to accommodate relatively large differential ground motion, at the present time there is no acceptable design procedure that would accommodate major differential fault movement in the reservoir embankment foundation. If the dam or dike is sited in a region (as defined by Federal Regulation) where such differential fault motion is credible, the dam or dike shall be assumed to fail.

4.4.6 Dynamic Stability and Performance. During an earthquake, large cyclic inertia forces are induced in an earth dam. These forces may be sufficiently large and may occur with sufficient cycles to produce excess pore water pressures or cause a reduction in shear strength of certain types of materials used in construction of an earth structure. Depending on the severity of the ground vibratory motions and the types of embankment materials, small to large permanent deformations of the embankment could occur during or after an earthquake. In loose saturated cohesionless soils complete loss of strength may occur, leading to failure of an earth structure. This same phenomena could also result from the effects of dynamic wave action, although the dynamic frequency characteristics of wave action make it a much less likely occurrence. Dams containing cohesive materials or well-compacted and graded materials generally suffered little or no damage as a result of strong ground shaking.[22] In assessing the safety of an earth dam during and after an earthquake (or other dynamic loading) the following factors should be considered:

(a) the magnitude and type of anticipated loading
(b) the degree of confidence in the method of analysis used in definition of material and design parameters.

The following minimum factor of safety is specified for the dynamic loading conditions listed in Section 4.4.5.

Condition	Minimum Factor of Safety
1	Precluded by siting criteria*
2	1.3
3	1.3
4	1.2
5	1.3

*Must evaluate based on the impact of a failure

4.5 Analytical Methods

4.5.1 Methods of Static Analysis. Various analytical methods for evaluating the static stability of an earth dam exist.[23, 24, 25, 26] The state of the art of static analytical methods is probably substantially more advanced than other facets of dam design, and for a given set of input data, most of these acceptable techniques will give results consistent with each other.

The method utilized shall be compatible with the anticipated mode of failure, dam cross-section and soil test data. The complexity of the method selected should also be consistent with the size of the structure. Whichever method is used, the Geotechnical Engineer shall state the justification for the method used.

Analyses shall be performed for the various loading conditions given in Section 4.4.3. The critical failure surface shall be presented for each case together with its corresponding factor of safety. The analyses shall take into consideration such variables as material types used for each zone of the dam, dam geometry, variability of soil properties (including location of phreatic surface and variation of pore pressures within the embankment).

4.5.2 Methods of Dynamic Analysis. Various methods of analysis are available for evaluating the seismic stability of an earth dam.[27 through 34] These may be classified as follows:

(a) pseudo-static methods
(b) simplified procedures
(c) dynamic response analyses.

Conventional pseudo-static methods of analysis are acceptable if the seismic coefficient selected appropriately reflects the geologic and seismologic conditions of the site and if the materials are not subject to significant loss of strength under dynamic loads. Values of shear strength[38] used in this type of analysis should reflect any anticipated loss of strength due to the postulated design earthquake.

Although pseudo-static methods of analysis are simple to use, they do not provide information on the magnitude of permanent deformations, which would develop within the embankment as a result of an earthquake. Where this information is of importance, methods (b) and (c) should be used. In recent years several simplified procedures have been developed based on Newmark's original concept of cumulative deformation.[27, 28, 29, 33, 35, 36] These simplified procedures may be used for earth dams constructed of materials that are not subject to significant loss of strength due to cyclic loading. (These include cohesive soils and well-compacted materials).

Dynamic response analyses using state-of-the-art methods shall be conducted for those dams located in highly seismic areas (or constructed of materials that could undergo significant loss of strength due to cyclic loading; i.e., hydraulic fill dams and tailing dams). Finite element techniques have been widely used for this purpose (although in recent years finite difference methods have also been developed.[30, 31, 32, 35, 37, 40] Appropriate dynamic material properties and ground motion parameters defined for the site shall be used in analyses. Considerable experience and engineering judgment are necessary in assessing the stability of an earth dam based on the results of a complex computer dynamic response analysis. In all cases, the results of such analyses shall be verified by general equilibrium checks.

5.0 Site Protection Earth Structures—Dams, Dikes, Breakwaters, Seawalls, Revetments

5.1 Scope

5.1.1 Purpose. The purpose of this Section is to describe criteria to be used as a guide in the design, evaluation and construction of those dams, dikes, breakwaters, seawalls and revetments classified as Seismic Category I. This standard is intended to identify factors to be considered in the construction of those structures and should in no way limit the investigation and analysis deemed necessary for determination of the suitability of such a structure and its site.

5.1.2 Use and Type of Structures. Dams, dikes, breakwaters, seawalls, and revetments are intended primarily to protect the nuclear plant site from hydraulic loads.

5.2 Site Investigations. A general discussion of site investigations can be found in Section 3.0. The investigation of sites for hydraulic protection earth structures shall be conducted in conformance with the following basic guidelines.

5.2.1 Waterfront Associated Parameters. These consist of natural shore and offshore zone characteristics, water motion characteristics, and shorefront behavior patterns. These shall be evaluated in conformance with Ref. 40. Investigation requirements shall be sufficient to clearly define the following basic waterfront associated parameters:

(a) coastal area and offshore profiles from the land bluff or escarpment for a sufficient distance offshore to define that depth of bed below stillwater level which can control the design wave form
(b) bathymetric and topographic contour maps of bed area sufficient to define the immediate influence of such features upon design of the structure
(c) natural protection features influencing water waves and flood
(d) exposure to storm attack
(e) characteristics of water waves, currents, surges and floods influencing the earth structure
(f) rate and composition of littoral transport and drift
(g) long-term stability of shoreline in terms of erosion or accretion rates.

Water and water level investigation requirements for design of the above structures shall include the following basic information:[11]

(a) stillwater or mean water level
(b) astronomical tide data
(c) seiche, wave setup and storm surge predictions
(d) design maximum flood elevation.

A determination of wind-generated water wave conditions as a basis for design shall include:[11]

(a) evaluation of all wave data applicable to the project site
(b) determination of the significant wave height and range of periods for the wave spectrum
(c) determination of the design depth of water at the structure
(d) determination of the design wave height, direction and condition (breaking, nonbreaking or broken) at structure site
(e) analysis of the frequency of occurrence of design conditions.

5.2.2 Geotechnical. Geotechnical parameters consisting of geologic, groundwater, foundation engineering and earthwork parameters shall be evaluated in conformance with Ref. 2.

Geotechnical investigation shall be sufficient to clearly define the following basic items:

(a) subsurface profiles along the length of the structure, and subsurface sections across the structure, prepared in a manner sufficient to define the spatial arrangement of soil and rock materials that could influence the structure design or safety
(b) detailed geologic and engineering descriptions of each material identified on the subsurface profiles and sections
(c) definition of physical properties, strength characteristics, and dynamic properties of the soil and rock materials defined on the subsurface profiles.

In establishing geotechnical site design parameters, if structures being considered are not at the nuclear plant site, then a literature review and search equivalent to that performed to develop nuclear plant site design parameters shall be undertaken to establish appropriate geologic, seismic, and natural phenomena.

Establishment of detailed geotechnical characteristics of subsurface materials shall include:

(a) surface geophysical surveys
(b) exploratory borings and excavations
(c) borehole geophysical surveys
(d) sampling of soil and rock materials
(e) the in-situ testing of soil and rock materials
(f) the laboratory testing of soil and rock materials.

Specific techniques and references applicable for each of the above outlined in reference (4) Special Procedures.

5.3 Materials. The investigation of soil, precast, armour, rock, rubble or stone for the construction of earth waterfront structures shall be sufficiently extensive to identify sources of adequate quality and volume for each of the required materials. Selection of a structure type and determination of the feasibility of the structures are dependent upon an adequate source and its associated quality. In general, Section 4.3 material selection requirements are equally applicable to site protection structures.

5.4 Design. Parameters to be established for the design and safety evaluation of dams, dikes, breakwaters, seawalls, revetments are generally the same as those given in Section 4.4.

5.4.1 Operating Conditions. Design conditions for site protection structures are generally those associated with extreme hydrological phenomena. However, normal operating conditions (which include erosion, weathering seepage or other normal operating phenomena that would affect performance of the protective structure) shall be considered in design.

5.4.2 Static Loading Conditions. The following conditions shall be considered for protective structures:

(1) During construction
(2) End of construction
(3) Design maximum flood evaluation as a hydrostatic load
(4) Load case where maximum design surcharge is present and water level is at its design minimum elevation.

5.4.3 Static Stability and Performance. Factors of safety for structural capacity should be based upon the ratio of available strength to applied stress or other load effects. The minimum factors safety for the static loading condition listed in Paragraph 5.4.2 shall be as follows:

Condition	Minimum Factor of Safety
1	1.1
2	1.3
3	1.2
4	1.5

In using these minimum recommended safety margins the Geotechnical Engineer should have a high degree of confidence in the reliability of values used for the following parameters:

(a) type and gradation of material
(b) thoroughness and completeness of field exploration and laboratory testing
(c) certainty of loading conditions
(d) degree of control and workmanship that can be assured.

5.4.4 Dynamic Loading Conditions. The dynamic force applicable to site protection structures are the same as those considered in Section 4.4.5.

5.5 Analytical Methods. The analytical methods applicable to ultimate heat sink structures are also applicable to site protection structures.

6.0 Site Contour Earth Structures—Retaining Walls, Natural Slopes, Cuts and Fills

6.1 Scope.

6.1.1 Purpose. The purpose of this Section is to describe criteria to be used as a guide in the design, evaluation and construction of those site contour control structures such as retaining walls, slopes, cuts and fills (classified as Seismic Category I). This standard is intended to identify factors to be considered in construction of those structures and should in no way limit the investigation and analysis deemed necessary for determination of the suitability of such a structure—or the effect such an earth structure would have on other nuclear plant structures.

6.1.2 Use and Type of Structure

6.1.2.1 Retaining Walls. A retaining wall is any permanent structural element built to support an earth bank that cannot support itself. It is used primarily to control site contours and may have specific application to construction of elevated or depressed roadways, erosion protection facilities, bridge abutments and retaining potentially unstable hillsides. Principal types of retaining walls considered in this standard include gravity walls, semigravity walls, cantilever walls, counterfort walls, buttressed walls, crib and bin walls, reinforced earth walls and anchored (or tie back) walls. The emphasis in this Section is on the design of earth structures used as retaining walls, and determination of loads on walls made of other materials.

6.1.2.2 Natural Slopes, Cuts and Fills. Natural slopes considered in this section

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are any landforms existing on, or adjacent to, the proposed site. A cut slope is any slope resulting from the excavation of in situ soils. Manmade fills are provided to maintain site grade. Slopes, cuts and fills covered by this specification are provided primarily to maintain site contours (and whose failure would adversely affect the function of any safety related nuclear plant structure).

6.2 Site Investigation. A general discussion of site investigation applicable to all earth structures is presented in Section 3.0.

6.2.1 Seismology and Geology. General seismic geology siting criteria are given in 10 CFR 100, Appendix A.[1] Various other references provide useful information on requirements that must be satisfied by a thorough seismologic and geologic investigation.[9, 10]

6.2.2 Hydrology. Earth structures used as retaining walls, slopes, cuts and fills are particularly sensitive to surface water erosion and groundwater level and movement. Such structures shall be designed to withstand historical and design basis flooding and precipitation in accordance with ANSI N 170.[11]

6.2.3 Geotechnical. In the construction of earth structures it is imperative that the structure cross-section, materials of construction and their gradation, zoning and placement be consistent with site geology and foundation conditions. Investigations shall be undertaken and sufficient information obtained so that the engineer can, with confidence, design a structure meeting those requirements. References discussing the required geotechnical investigations in considerable detail should be consulted.[11, 12, 13, 14, 15, 16, 17, 20]

Since natural slopes and cuts consider the use of in situ materials, available literature and information concerning the foundation geology of the soils (and of rocks on the site) shall be consulted. Past records of construction in the area and old well logs shall also be examined. Airphoto interpretation and site reconnaissance should be completed to reveal old slide scarps or other evidence of slope movements. Cross-sections and profiles of the slope should be made in sufficient quantity and detail to represent the slope and foundation conditions.

6.3 Materials. Section 4.3 material selection requirements are equally applicable to retaining walls, slopes and fills.

6.4 Design

6.4.1 Design Parameters. Parameters to be established for the design and safety evaluation of retaining walls, natural slopes, cuts and fills shall include the following:

(a) a geotechnical profile along the entire length and across the structure at intervals not to exceed 250 feet, which is adequate to serve as a basis for design
(b) the potential for ground surface rupture or displacement due to geological factors
(c) ground surface acceleration value for the SSE
(d) properties of available cast shapes, rubble, stone, rock, in situ and filter materials used for construction of the structure
(e) cross-sections showing structure geometry and composition of materials
(f) liquefaction potential of the earth structure and its foundation under (a) the SSE and (b) hydrodynamic changes in effective stress caused by the maximum design event
(g) stability of the structure and its foundation under hydrodynamic and surcharge force systems associated with maximum design event
(h) hydrological parameters shall be in accordance with ANSI N 170.[11]

6.4.2 Operating Conditions. Operating conditions for contour control structures will vary according to the purpose, location and other conditions unique to the plant being considered. These conditions may influence the design of ancillary facilities. The Geotechnical Engineer shall consider all normal operating conditions in design of the structure, as well as anticipated transients, abnormal and extreme environmental conditions considered as design basis during the life of the structure.

6.4.3 Static Loading Conditions. The following conditions shall be considered for contour control structures:

(1) During construction
(2) End of construction
(3) Maximum design surcharge to include any loading above grade by earth, material, structure, equipment and vehicles for design against sliding
(4) Load condition 3 coincident with most disadvantageous ground water design level
(5) Maximum design surcharge to include any loading above grade by earth, material, structure, equipment and vehicles for design against overturning
(6) Load condition 5 coincident with most disadvantageous ground water design level
(7) Design maximum flood and precipitation as a hydrostatic load.

6.4.4 Static Stability and Performance. Factors of safety for slope stability studies should be based upon the rate of available strength to applied stress or other load effects. The minimum factors of safety for the static load conditions listed in Section 6.4.3 shall be as follows:

Condition	Minimum Factor of Safety
1	1.3
2	2.0
3	1.5
4	1.3
5	2.0(*)
6	1.8
7	1.0

*For foundation failure by bearing in clay use a F.S. of 3.0. In using these minimum recommended safety margins the Geotechnical Engineer should have a high degree of confidence in the reliability of the values used for the following parameters:

(a) type and gradation of material
(b) thoroughness and completeness of field exploration and laboratory testing
(c) certainty of loading conditions
(d) degree of control and workmanship that can be assured.

6.4.5 Dynamic Loading Condition. The effects of earthquake-induced forces, dynamic surcharge loadings and the dynamic effects of the Design Maximum Flood and Precipitation[11] must be considered. The postulated loading conditions due to dynamic loads to be evaluated are as follows:

(1) Failure due to disruption of structure by major differential fault movement due to a SSE
(2) Slope failure induced by SSE vibratory ground motion
(3) Sliding of the earth structure on weak foundation materials or materials whose strength may be reduced by liquefaction
(4) Failure due to dynamic surcharge load effects if any
(4) Failure due to dynamic loads associated with the Maximum Design Flood or Precipitation.

6.4.6 Dynamic Stability and Performance. During an earthquake, or in response to other dynamic load phenomena, large cyclic forces may be induced in a slope or fill. These forces may be sufficiently large and may occur with a sufficient number of cycles to produce excess pore water pressures or reduction in shear strength of certain types of materials used in construction of an earth structure. Depending on the severity of the ground vibratory motions and the types of embankment materials, small to large permanent deformations of the embankment could occur during or after an earthquake. In loose saturated cohesionless soils complete loss of strength may occur, leading to failure of an earth structure. This same phenomena could also result from the effects of dynamic wave action although the dynamic frequency characteristics of wave action make it a much less likely occurrence. Structures containing cohesive materials or well-compacted and graded materials generally suffered little or no damage as a result of strong ground shaking.[22]

In assessing the safety of an earth structure during and after an earthquake—or other dynamic loading—the following factors should be considered:

(1) The magnitude and type of anticipated loading
(2) The degree of confidence in the method of analysis used and in the definition of material and design parameters.

The following minimum factor of safety is specified for the dynamic load conditions listed in Section 6.4.5:

Condition	Minimum Factor of Safety
1	Precluded by Siting Criteria*
2	1.3
3	1.3
4	1.3
5	1.2 (general) 1.0 (local)

*Must evaluate based on the impact of a failure

6.4.7 Other Design Considerations. Other considerations that may affect the design shall be investigated as necessary:

(1) Removal of lateral support including action of:
   (a) erosion by streams, rivers, etc.
   (b) waves and longshore tidal currents
   (c) subaerial weathering, wetting and drying and frost action.
(2) Removal or creation of new slope by rock fall, slide or subsidence (faulting).
(3) Subterranean erosion, solution carbonates, salt, gypsum, and collapse of caverns, subsidence of mine areas, dispersive soils.
(4) Overloading of weak underlying soil layer(s) by fill.
(5) Overloading of sloping bedding planes.
(6) Oversteepening of cuts in unstable soil or rock and undercutting of steeply adverse dipping bedding planes.

6.4.8 Performance Criteria. The performance of any slope must be judged on the following basis:

(1) Downslope Movements. Downslope movements, whether for natural or manmade slopes, shall not interfere with the ability of the plant to perform its safety functions. This necessitates consideration of the proximity of the slope to Class I structures and the specific function of the slope, if any. (The definition of slope failure is dependent on these conditions).
(2) Erosion and Undercutting. Erosion and undercutting of the toe of the slope shall be controlled so that they will not affect the overall stability or function of the slope.
(3) Creep. If the plant and/or adjoining facilities are sited on a slope, creep movements of sufficient magnitude can constitute a failure, as well as general massive instability of the slope. The potential for creep and the magnitude that can be tolerated shall be evaluated.

6.5 Analytical Methods and Procedures

6.5.1 Retaining Walls. Once the soil types and design parameters have been established, the type of retaining structure can be selected. Generally the foundation conditions, the height of wall, or the expected lateral load narrows the selection process considerably. Typical dimensions and guidelines for sizing the proportions of retaining structures are given in various foundation texts.[41, 42, 43] The structural adequacy of the individual members should be determined by the Geotechnical Engineer or Engineer based on the imposed loads, using applicable Standards.

6.5.1.1 Earth Pressure Computation. As defined previously, earth pressures acting on the wall are computed using appropriate soil properties (usually strength) and available earth pressure theories. The design magnitude and distribution of these pressures should also take into consideration the type of backfill and its characteristics and drainage provisions, and the method and direction of compaction. Clayey soils can produce high earth pressures and should be avoided if possible. Free draining clean granular soils generally result in lower horizontal earth loadings.

For conventional retaining walls, convenient empirically established design charts are available for different types of backfill.[44] These curves have also been reproduced in most geotechnical engineering text books that cover this subject.

6.5.1.2 Stability Against Sliding. The pressures acting on the wall tend to cause the wall to slide along its base. The stability against sliding shall be analyzed by summing up the horizontal forces. In practice, conservative factors and judgment must be applied to the analyses by neglected small passive resistance at the toe and accounting for disturbances during construction by applying reduction in strength along the base. A foundation key sometimes must be provided to attain a higher factor of safety.

6.5.1.3 Stability Against Overturning. The pressures acting on the wall also tend to overturn the wall about its toe. The wall shall, therefore, be checked so that the structure will be safe against overturning. In making this evaluation, all of the short and long term loads described previously shall be considered and used where appropriate, both in front of and behind the wall. Also, possible future excavations, erosion, uplift, liquefaction and other potential undesirable influences should be considered, especially with respect to the soils in front of the wall (i.e., the resisting forces).

6.5.1.4 Foundation Stability. Instabilities can develop due to a soil-bearing failure of the wall base. In addition to bearing, the expected settlements should also be studied to be sure that they will be within acceptable limits.

6.5.1.5 Overall Stability. Analyses should be performed to assure that the overall wall, the weight of the fill behind the wall and any upslope or downslope unstable driving forces will not cause a deep-seated bearing or sliding failure extending beneath the entire structure.

6.5.2 Analysis of Slopes

6.5.2.1 General Provisions

(1) Methods, A description of the method(s) of slope stability analysis used shall be provided. A definition(s) of factor of safety shall be stated. The method of analysis should reflect the anticipated mode of failure and methods of analysis for flow slides, sliding block (wedge), rock slides and lateral spreading may be required. The critical failure surface should also be presented.

(2) Design Parameters. Design soil parameters relating physical characteristics, strength, consolidation and chemical properties shall be evaluated for each statigraphic unit composing the slope for both static and dynamic conditions. A description of the groundwater level and flow conditions, if any, shall be given.

6.5.2.2 Static Analysis

(1) Appropriate methods of analysis as described in the literature are required to determine the most critical failure surface in the slope. Methods such as Taylor's friction circle method,[45] Bishop's method of slices,[46] Lowe and Karafiath,[47] Spencer's method,[48], Rendulic's logarithmic spiral,[49] or the irregular failure plane method of Morgenstern and Price[50] are all applicable limit equilibrium analyses.

(2) These analyses shall consider variables such as: slope shape, soil stratification, variability of soil properties, driving and resisting forces acting on the slope and variation of pore pressures within the slope.

(3) The influence of adverse conditions (such as floods, freezing, change in ground water conditions, rapid drawdown, steady seepage and their possibility of occurrence) shall be investigated.

(4) The theoretical assumptions of any particular method of analyses should be reviewed to determine their effect on the resultant failure surfaces and their factors of safety.[51 and 52]

6.5.2.3 Dynamic Analysis

(1) Various methods of analysis are applicable for evaluating the seismic stability of natural slopes. These include the following:
(a) pseudo-static methods
(b) simplified procedures for cal-

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culating earthquake-induced deformations (for example, Newmark's Cumulative Deformation Procedure)

(c) Dynamic Response Analysis

(2) For slopes comprised of clayey materials, materials that are compacted and moderately dense, and materials that undergo little or no loss of strength due to cyclic loading procedures (a) and (b) above provide adequate methods of analysis. For loose to moderately dense saturated cohesionless soils and materials showing a significant loss of strength due to cyclic loading, a dynamic response analysis is required. Two-dimensional finite element analyses using equivalent-linear, strain-compatible dynamic properties—together with the results of laboratory stress-controlled cyclic triaxial or cyclic simple shear tests—are often used for this purpose.

An appropriate acceleration time history is required for use as an input motion to the model when conducting a dynamic response analysis.

(3) In the above analyses, the equivalent linear model representation of shear modulus vs strain relation is suggested.

(4) The forces obtained from the above analyses may be considered as analogous pseudo-static forces that can be incorporated as part of the loading applied to the static method of analysis.

(5) Special considerations should be made for the possibility of liquefaction of slope materials if site and laboratory investigations indicate susceptible deposits. Effects of lateral spreading should be considered.

6.6 Specific Provision

6.6.1 Protection of Slopes

6.6.1.1 Stabilization of Potential Slide Areas. Provisions should be made for stabilization of slopes against mass sliding or other movements in all potentially unstable areas under consideration. Such provisions could include:

(1) Reduction of loads—flattening of natural slopes, lowering ground water level by means of internal drains, or removal of soil at the top of the slide area.

(2) Reduction of excess pore water pressures by improving surface and internal drainage.

(3) Increase of resisting forces by berms or earth buttresses at the toe.

(4) Use of structural support, such as retaining walls, earth or rock anchors, or sheet piling.

(5) Special methods of soil stabilization such as a cement, lime, flyash or asphalt stabilization, or densification by preloading or vibratory methods.

6.6.1.2 Special Provisions Regarding Erosion

(1) Soil Erosion. Provisions should be made to minimize soil erosion and creep on natural and cut slopes by maintenance of sufficient vegetation on the slopes, paved ditches, use of rip rap, to prevent gullying and other erosive features that could affect the stability of the slope. Erosion and poor drainage are frequently the cause of failures.

(2) Wave Protection. Slopes affected by wave action should be protected by rock rip rap or equivalent.

(3) Chemical Interaction. Chemical tests shall be performed to determine the interaction between the chemistry of the soil, pore water, and eroding water for the evaluation of dispersion potential.[53, 54, 65]

7.0 Inspection, Instrumentation and Monitoring for Construction

7.1 Introduction. Often, the design and analysis of earth structures based on nominal material properties and design assumptions alone are inadequate. Coupled with design and analysis must be:

(1) Verification of design assumptions and material parameters
(2) Evaluation of the inter-relationship between construction methods and analytical treatment
(3) Verification testing to assure compliance with construction specifications based on the design
(4) Monitoring of the earth structures both during and subsequent to construction.

Because of the need for a continuity from design and analysis through actual construction, this section is included in this standard.

It is intended that this section provide for the testing of materials and assure a continuity of service throughout the entire geotechnical engineering program. It is recommended that these functions be completed so that all steps in the process from initial site feasibility studies to operation of the plant be completed—and that a mechanism for changes in design be provided if required.

It is recommended that a qualified onsite geotechnical engineer be present at all times during earthwork activities to provide continuous observation.

7.2 Design Verification. This function is defined as the verification of assumptions pertinent to the design analysis.

Because of the uncertainties inherent in subsurface exploration, it is necessary that field observations show the actual conditions agree with those assumed in design. This includes, but is not limited to, observation of areas that have been excavated of existing geologic conditions or the review of bearing conditions prior to backfill or construction. The conditions actually encountered shall be assessed for their effect on design. Design verification should be performed by the operation that was responsible for the original geotechnical design and analysis. The actual conditions should be reviewed to determine how the work is affected and the design modified as appropriate. It is recommended that one organization maintain overall responsibility for verification activities.

Coordination of the on-site verification shall be required so that verification can be accomplished at that point in site construction which provides that best opportunity for observation. This coordination shall be provided by the on-site Geotechnical Engineer to the necessary organizations.

Techniques and test methods used during design verification, such as plate load tests or rock soundness testing, shall be in accordance with methods accepted in the geotechnical profession.[3, 4, 13, 14]

7.2.1 Geologic Verification. When subsurface conditions are revealed during excavation, it shall be verified that the exposed soil and rock materials are consistent with that shown on the boring logs and profiles. Included shall be verification of the stratigraphy, classification and geologic mapping. Particular interest shall be made to such geologic features as faulting. Also, as required, verification of bearing areas shall be made to determine adequacy. For rock verification, this could include rock soundness determination, while for soils it could include plate load tests. Consideration shall also be given to the performance of additional laboratory testing or to in situ testing (to verify strength parameters used in the analysis). Examples of this would be in situ direct shear tests on soil or rock or obtaining block samples for laboratory testing.

7.3 Construction Specification

7.3.1 Introduction. The analyses and design of earth structures by the Geotechnical Engineer are based upon certain design assumptions, including strengths, materials properties and other parameters. The Geotechnical Engineer shall provide Construction Plans and Specifications.

7.3.2 Information Addressed in the Construction Specifications. As a minimum, the following areas should be addressed in the Construction Specification.

7.3.2.1 Materials. Materials consistent with the design should be specified, and procedures defined for verification of their use, by visual as well as laboratory tests.

7.3.2.2 Placement and Compaction. Placement and compaction procedures and lift thicknesses should be specified or com-

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paction criteria established as part of the specification. Test fills may be required. Care should be taken to assure that over-compaction is not achieved as this may tend to increase lateral stresses, possibly affecting stability or integrity of the structure.

7.3.2.3 Drainage. Most earth structures have, as a minimum, some type of drainage system. Detailed drawings of drainage requirements shall be included in the specifications and, for clarity, shall include such details as pipe size and type, filter material types and layer thickness and pipe slope requirements.

7.4 Construction Process Review. In addition to the preparation of a Design Specification, it shall be the responsibility of the Geotechnical Engineer that performs the design and analysis of the earth structures to review the construction procedures proposed by the constructor. The review shall be performed by the Geotechnical Engineer prior to beginning of construction on a particular activity. The purpose of the review shall be to provide another step in the continuity of geotechnical service by assuring that necessary assumptions made in the design and analysis are not invalidated by the construction method or sequence so as to affect performance of the structure. During construction, it shall be the responsibility of the on-site Geotechnical Engineer to observe the actual construction process to verify that the construction method or sequence is properly performed. In addition, it shall be the responsibility of this individual to provide verification during construction of those steps which could affect performance.

7.5 Verification Testing. Verification testing shall be performed using standard or specified procedures. Verification testing is the in-process inspection and testing that is performed to verify that the Construction Specifications and Procedures are satisfactorily fulfilled. This activity shall be supervised by a Geotechnical Engineer. In general, for safety-related earthwork, verification testing is associated with excavation, backfilling or the construction of embankments or retaining structures. Discussed below are several typical activities.

7.5.1 Excavation. Excavation is the process of removing soil or rock so that a structure may be founded below existing grade, or the removal of undesirable materials which will, in turn, be replaced with material of improved properties commonly referred to as engineered backfill.

As a part of excavation work, a dewatering system may be required to lower the groundwater table. Responsibility for the design and installation of a dewatering system, including the possible conduction of a pump test, shall be defined. As part of a dewatering system, piezometers or other devices to monitor the groundwater level shall be installed to verify performance of the system. A program for the periodic reading of these instruments shall be established. Suspended solids in water removed shall be measured periodically to verify that soil material (fines) are not being pumped. Inspection services for excavation work shall be dependent upon the scope of the work and whether it is intended to reuse excavated materials. For excavation materials that are classified as "spoil" only, inspection services shall be required to verify that excavation has been completed to the specification limits, that all unsuitable material is removed, and all "spoil" material is disposed of in areas designated for this use. If excavated materials are to be separated as "spoil," or for reuse, inspection shall be required in the excavation to visually classify materials for either reuse or disposal. Disposal areas that are used for materials to be later reclaimed shall be inspected to prevent inadvertent dumping of "spoil" material. For excavation performed to design slopes, inspection services shall be provided to assure that slopes, benches, etc., conform to project specifications. Observations should be made by the Geotechnical Engineer to verify during excavation that the material actually encountered is found to be as expected based on borings. The presence of extraneous or unexpected material may require exploration and evaluation.

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7.5.2 Stockpiling. The stockpiling of materials shall include soils and rock from excavation work as discussed above or the bringing on-site of materials for later use. Stockpiling of excavated materials shall include the inspection mentioned above. For materials brought on-site, an inspection shall be conducted upon arrival to verify that they meet project specifications. Inspection in stockpiling areas shall be provided as required.

7.5.3 Borrow Areas. When a borrow area is in use, inspection services are required to verify that materials are removed as specified. Visual classification of soils shall be supplemented by periodic verification testing—depending on how the material is specified. This testing is not to be misconstrued as being replacement for backfill testing discussed below.

7.5.4 Test Fills. To determine an adequate technique for material placement so that the Construction Specifications and Procedures are met, a test fill may be required by the Geotechnical Engineer. The test fill should consider material type, lift thickness, type of equipment (if this is a variable), number of passes, and the means for reducing or increasing moisture content if necessary. Inspection services shall be provided to document different techniques employed and to sample the test fill for gradation, moisture content and in situ density and lift thickness for comparison with project specifications.

7.5.5 Backfill Verification. Backfill verification shall include testing and continuous visual inspection services provided to assure the placement of earth and/or rock materials in accordance with project specifications. Included are the placement of materials for foundation-bearing areas, backfilling up to grade around structures, placement of materials for constructing embankments and dikes (including cores and drains) and the placement of materials for erosion protection such as rip rap. Backfill verifications shall include continuous visual inspection to assure proper lift thickness, placement of materials in zones, and the number of passes by compaction equipment. Testing services shall be conducted at specified intervals, usually in terms of quantity of material placed or a minimum daily testing quota. Such testing shall be performed to verify specifications and may include gradation, moisture content, and in-situ density and compaction characteristics.

7.5.6 On-Site Laboratory. For a quality assurance/quality control testing associated with earthwork verification, an on-site laboratory should be provided and staffed by the organization responsible for this work.

The equipment used in verification testing shall be as described in the testing standard adopted for use. Equipment shall be calibrated as discussed in Section 7.7.2.

7.6 Performance Monitoring. Performance monitoring is conducted to assure that the completed work satisfies design assumptions. Included may be plate-load tests, pressuremeter tests, settlements monitoring, groundwater monitoring, geophysical methods, and in-situ densification verification.

Performance monitoring shall be implemented so that sufficient data are obtained, including base-line data prior to construction if necessary. Base-line data should include testing conducted to obtain in-situ densities prior to a densification program. Monitoring shall be extended beyond the construction period as required to verify design assumptions. The results of such monitoring should be available to, and evaluated by, the organization responsible for design and analysis. It is recommended that the concept and implementation of performance monitoring be stipulated by the design organization. The purpose of the performance monitoring is not to verify work as construction progresses, but rather to provide verification that design assumptions have been met. Performance monitoring shall be conducted in accordance with accepted test procedures, with calibrated equipment, and at adequate testing frequencies.

Discussed below are several general areas to be considered in developing a performance monitoring program. Additional programs besides those discussed

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below should be considered as required.

7.6.1 Monitoring for Seepage or Groundwater Control. Performance monitoring may be required to verify design assumptions associated with seepage or groundwater control. This could include: dissipation of excess pore pressures due to embankment or dike construction, assumed pressure distributions, or the cutoff of seepage by grouting or slurry walls.

7.6.2 Deformation Monitoring. Deformation monitoring shall include both placement of monuments to serve as reference points and installation of deformation points to define movement of subsurface strata or a structure. Deformation monitoring shall include a defined surveying program with adequate installation or benchmarks. Vertical movements may be monitored to determine either settlement or heave of a structure or by establishing them at desired depths in the subgrade. The reference benchmarks shall be located so that they are unaffected by construction activities. Lateral movements shall be monitored where required using instruments such as inclinometers. Lateral movement measurements should be considered for excavated slopes and retaining walls.

7.6.3 Stress and Load Measurement. Instrumentation may be installed to verify design assumptions associated with embankment or retaining walls construction or structure-bearing pressures. This could include: strut load measurement (both for temporary or permanent bracing for retaining walls), loads in tie backs for retaining walls, total stresses due to placement of an embankment to determine end-of-construction conditions, monitoring of bearing pressures (both during and after construction) to verify assumed contact pressures and predicted settlements history.

7.7 Quality Assurance/Quality Control. Organizations performing verification, monitoring or inspection services shall have established Quality Assurance/ Quality Control programs which satisfy requirements of 10 CRF 50, Appendix B as it pertains to their work. Responsibility for the Quality Assurance/Quality Control program shall be with the organization performing that work.

ANSI N 45.2.5—1978 Supplementary Quality Assurance Requirements for Installation, Inspection and Testing of Structural Concrete, and Structural Steel, Soils and Foundations During the Construction Phase of Nuclear Power Plants, QA-76-5, (8) is pertinent.

7.7.1 Qualification of Personnel. Personnel involved in the verification testing program and support pesonnel for the performance monitoring program shall be qualified as needed for their activities in accordance with ANSI N 45.2.6.

Geotechnical engineers and geologists involved in design verification, the construction review process and performance monitoring shall be qualified to perform their assignments. Qualification of these personnel shall be demonstrated by their education, experience and appropriate licensing.[55]

7.7.2 Requirements for Equipment Calibration. Calibration shall be required for all instruments or test equipment that provide quantity measurement used in design verification, verification testing or performance monitoring programs. Calibration requirements shall include initial acceptance criteria, frequencies for recalibration and tolerance limits. Tolerance limits shall be established based on the instrument and the quantity being measured.

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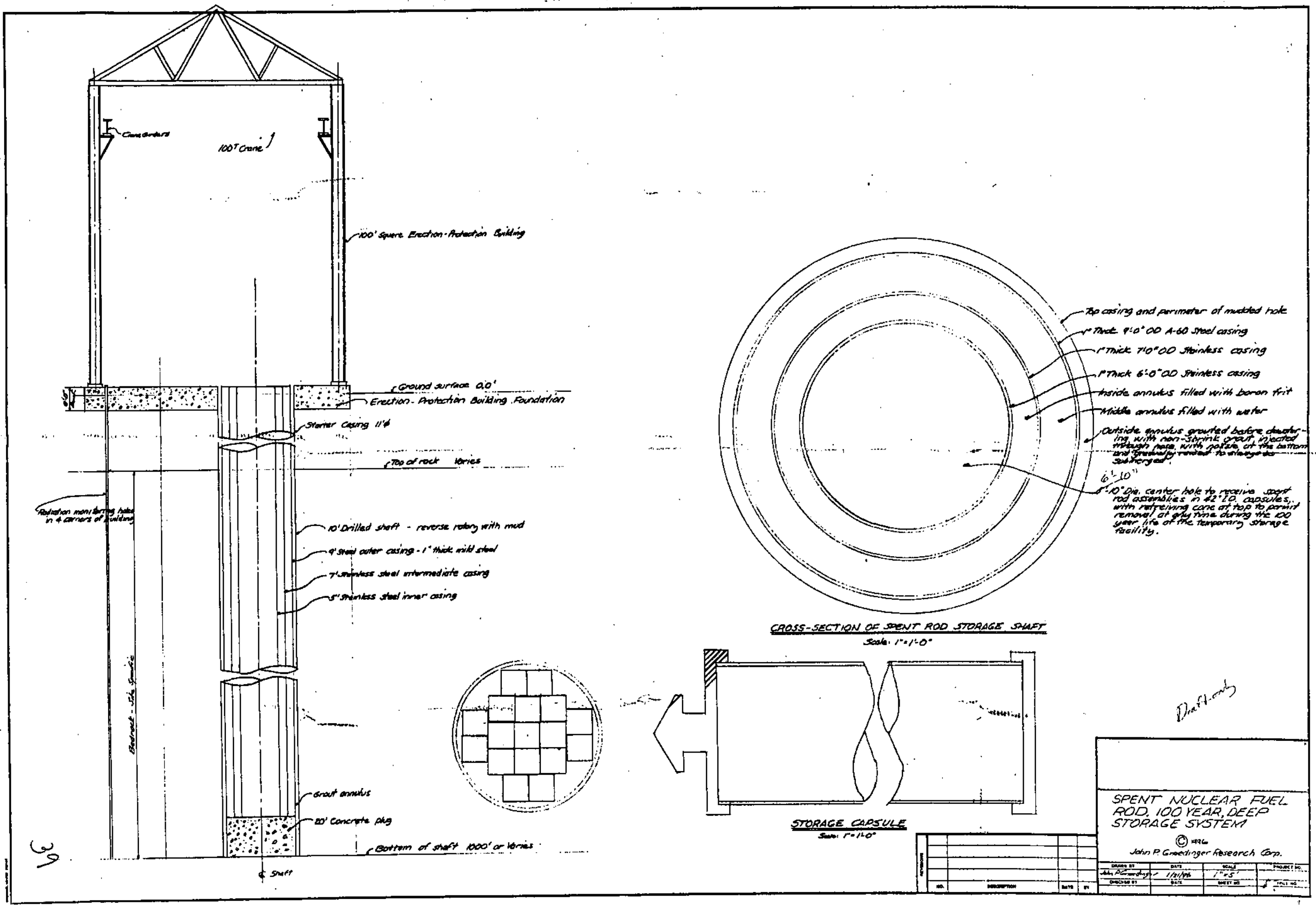

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